공학석사 학위논문

Assessment of applicability to estimate shear strength parameters of weathered rock by field tests

현장 시험을 통한 풍화암 지반의 강도 정수 산정의 적용성 평가

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이 논문을 공학석사 학위논문으로 제출함

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2018년 2월

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Abstract

Assessment of applicability to estimate shear strength parameters of weathered rock by field tests

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A weathered granite is distributed broad area in Korea. It is used base of support for a deep foundation in Korea. However, determining of material properties of the weathered granite is very difficult. Sample recovery of weathered granite is hard because it is easily broken and losing its rock structure during coring or drilling. Present research try to determine reasonable shear strength parameters, cohesion and internal friction angle of the weathered granite in Korea by field tests, pressuremeter test (PMT) and borehole shear test (BST). Moreover, present study try to assess applicability of these. Present study estimate shear strength parameters from PMT, BST, lab tests. The whole test results from PMT, BST, lab tests, and previous research are compared. Tests results performed in this study are not corresponded within themselves. Lab test results are considered as a true value for comparison of each results.

Keywords: Weathered rock, Shear strength parameter, Pressuremeter test, Borehole shear test

Student Number: 2016-21257

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Chapter 1 Introduction

1.1 General

A weathered granite is distributed broad area in Korea. It is used base of support for a deep foundation in Korea. The weathered granite has wide range of material properties because it is a transition stage of rock to soil. However, determining of material properties of the weathered granite is very difficult.

Lab test should require an undisturbed sample but sample recovery is not easy for weathered granite because it is easily broken and losing its rock structure during coring or drilling. Field test is also not easy to adapt because of its great strength and stiffness. Generally, Standard Penetration Test (SPT) is used to assess for the weathered granite in Korea. Nevertheless, result of SPT on the weathered granite shows little change of embedment depth per 50 blows comparing with great change of its strength. Furthermore, a mechanism during blows is not clear because SPT is suitable for the sandy soil, but the weathered granite still has rock structure. Therefore, in the past in Korea, the material properties of the weathered granite is underestimated to the same as those of very dense sand.

After 90s, there are many researches on resistance of weathered granite based on a research related to resistance of Intermediate Geo-Material (IGM) (O'Neil & Reese, 1999; Lee et al., 2005; Kim et al., 2006; Ahn, 2013). Most of researches in Korea are simply borrowing the IGM design theory proposed

in U.S and adopting it to weathered granite in Korea. Those researches did not reflect mechanical characteristics of the weathered granite. Therefore, a research for the reasonable estimation of the weathered granite is inadequate in current situation.

The weathered granite is generated by weathering process from an under parent granite. The parent granite experience several weathering process such as destressing from uplifting of base rock, growth of fracture, decomposing of its components. Then, the weathered granite has different physical, chemical characteristics from the parent granite. As the weathering process goes further, both rock mass and intact rock properties weaken. Many previous studies, which try to quantitatively determine degree of weathering and to correlate it with mechanical properties of the weathered granite, have been performed (Gupta & Rao, 1998; Begonha & Braga, 2002; Lee & Chang, 2003; Sun et al., 2006).

Present research try to measure reasonable strength parameter cohesion c' and internal friction angle ϕ' of the weathered granite in Korea by performing field tests such as pressuremeter test (PMT) and borehole shear test (BST).

Pressuremeter is a cylindrical probe that has an expandable flexible membrane designed to apply a uniform pressure to the walls of a borehole (Clarke, 1995). PMT is usually performed to measure deformation modulus of geo-material. However, through some special methods, it can be used to calculate the strength parameters of the ground. Borehole shear test is a field direct shear test. Shear plate is pulled up under confining pressure on the

target depth. This test result gives strength parameters directly under Mohr-Coulomb failure criteria.

1.2 Outline

This study is divided into five chapters.

Chapter 2 provides backgrounds such as definition of the weathered rock in Korea, distinctive characteristics of the weathered rock from other geo-material, and several theories related to applied filed tests.

Chapter 3 describes a site where the field tests performed. In addition, ground information is explained by using conducted standard penetration test (SPT). Calculating the shear strength parameters of the weathered rock based on the test results.

Chapter 4 discuss about the shear strength parameters estimated in chapter 3. Limitations and improvement plan of each field test are discussed.

Chapter 5 summarizes the result of this study.

Chapter 2 Background

2.1 Classification of the weathered rock

2.1.1 Practical classification system in Korea

For limiting research scope, Definition of the weathered rock needs to be assured. Here are practical steps for the ground classification procedure in Korea. Standard penetration test (SPT) is performed until a soft rock layer appears. Determining soft rock layer depends on several factors such as sounds of the drilling, colours and unconfined compressive strength (UCS) of the recovered cores. Then, the weathered rock layer is determined after the depth that shows 10 or 15 cm embedment per 50 blows on SPT and before the depth of the soft rock layer.

As already mentioned before, the weathered rock is an intermediate geo-material. The weathered rock can be considered as a very stiff or dense ground in the aspects of soil layer and as a very weak rock mass in the aspects of rock layer. Therefore, the classification system for each geo-material needs to be checked.

2.1.2 Soil and rock classification system

To classify the soil type, there are two typical classification systems. They are Unified Soil Classification System (USCS) (ASTM D2487-11) and AASHTO classification system. Both USCS and AASHTO system classify the soil based on the laboratory tests such as sieve test and Atterberg limit test. Those soil classification systems are not suitable for the weathered rock. The weathered rock in nature still has a rock structure but the recovered sample used for the lab tests lose its structure because of sample disturbance.

Therefore, soil classification with field test is proper for the weathered rock layer. SPT and Cone penetration test (CPT) are usually used to classify the soil types. CPT is generally used to classify the clayey ground because cone penetration needs to apply heavy reaction force to make the cone penetrate the stiff ground. On the other hand, in SPT, an embedded depth is measured during repetitive blows. It can be easily applied to very hard ground without special treatment. Korea Expressway Corporation (2009) and Korea Rail Network Authority (2011) suggested that the soil shows layer less than 15 cm embedded depth per 50 blows on SPT as the weathered rock layer. Seoul Metropolitan Government (2006) suggested similarly but 10 cm embedded depth as the weathered rock layer.

Meanwhile, irrespective of soil classification system, there are many kinds of rock classification system such as rock mass rating (RMR), Q-system, rock quality designation (RQD). They have difference little bit each other but they have similar concept that put the information about strength of intact rock,

ground water, discontinuity together and classify the rock layer. The biggest different concept of rock classification system with soil classification system is the concept of rock mass and intact rock. Rock mass consist of intact rock and discontinuities. When considering the soft rock or hard rock, characteristics of discontinuities in those of rock can be observed by recovered core. However, in the case of the weathered rock, it is very hard to get a good quality core then it is difficult to observe discontinuities clearly. ASTM D6032-17 said *RQD is a modified core recovery in which the ratio of length of core recovered to the total length drilled is modified such that only the length of the pieces of sound core that are equal to or greater than 100 mm in length. The term sound core that is unweathered to moderately weathered and has sufficient strength to resist hand breakage.*

Therefore, when determining the weathered rock layer, classification system used for rock mass is not proper because of the difficulty of measuring characteristics of discontinuities. However, here is another index for the classification of the weathered rock layer. These are shear wave velocity (V_p) and unconfined compressive strength (UCS) of the recovered core. Korea Expressway Corporation (2009) suggested 3.0 ~ 3.5 km/s of V_p and 0.2 ~ 60 MPa of UCS, Korea Rail Network Authority (2011) suggested lower than 3.5 km/s of V_p and lower than 5 MPa of UCS and Seoul Metropolitan Government (2006) suggested 1.0 ~ 2.5 km/s of V_p and lower than 10 MPa of UCS.

After 90s, new concept called Intermediate Geo-Material (IGM) have suggested by O'Neill & Reese (1999). IGM is essentially a type of soil, which

follows classification such as cohesionless and cohesive. The cohesionless IGM defined as 50 ~ 100 blows of SPT N value which means lager than 50 blows per 30 cm embedment depth and lower than 50 blows per 15 cm embedment depth. Table 2.1 summarize the criteria of the classification system for the weathered rock.

Table 2. 1 The criteria of the classification system for the weathered rock

Proposer	**Criteria**	
	V_p (km/s)	**UCS (MPa)**
Korea Expressway Corporation (2009)	3.0 ~ 3.5	0.2 ~ 60
Korea Rail Network Authority (2011)	$\leq$ 3.5	$\leq$ 5
Seoul Metropolitan Government (2006)	1.0 ~ 2.5	$\leq$ 10
O'Neil & Reese (1999)	50 ~ 100 (SPT N value)	

2.1.3 Geological classification of weathering

Chapter 2.1.1 and 2.1.2 shows classification of the weathered rock for the engineering purpose. However, weathering of the rock material have been geological issue. Hamrol (1961) has attempted to quantify the degree of chemical weathering and weatherability of rocks on the basis of variation in the saturation moisture content. Lliev (1967) in turn stressed the need for a quantitative assessment of the degree of weathering in rock, which could be based on those physico-mechanical properties. He suggested a coefficient of weathering from ultrasonic velocities in the rock material. Cottiss et al. (1971) listed up the standard tests for the classification and characterization of intact rock material and for the determination in the laboratory of engineering design parameters. Irfan and Dearman (1978) correlated mechanical properties and weathering indices of granite. Especially, Lee & de Freitas (1989) organized geological descriptions and classification system for weathered granite in Korea and he demonstrated typical weathering profile of Korean granites.

The previous studies divided weathering grade of the rock into six groups, which are Fresh, Slightly weathered, Moderately weathered, Highly weathered, Completely weathered, Residual soil. The weathered rock phrased in this study ranged over Highly weathered to Completely weathered zone. Figure 2.1 summarized weathering scheme for granitic masses (Lee & de Freitas, 1989).

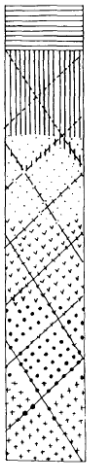
1. Typical weathering profile of Korean granites	2. Classification			3. Description	
	Zone	Term	Abbre-viation	Distribution of rock material within joint-bounded block and its grade	Simplified expression[1]
	VI	Residual soil	RS	Most material is RS grade	$RS^{C_R}_{P_R}$ (HP 0.3) (SL 4)
	V	Completely Weathered	CW	Most material is CW grade	$CW^{C_C}_{P_C}$ (SL 2)
	IV	Highly Weathered	HW	Inner material is HW grade; outer material is HW or CW grade, occasionally RS grade	$HW^{C_H}_{P_S}90\ CW^{C_C}_{P_C}10$ (Sh 18) (SL 1)
	III	Moderately Weathered	MW	Inner material is MW grade; outer material is MW or HW grade, occasionally CW grade	$MW^{C_M}_{P_S}90\ HW^{C_H}_{P_M}10$ (SH 42) (SH 22)
	II	Slightly Weathered	SW	Inner material is SW grade; outer material is SW or MW grade.	$SW^{C_S}_{P_S}95\ MW^{C_M}_{P_M}5$ (SH 55) (SH 45)
	I	Fresh	F	Inner material is F grade; outer material is F or SW grade.	$F^{C_F}_{P_F}95\ SW^{C_S}_{P_F}5$ (SH 60)(SH 57)

Figure 2. 1 weathering scheme for granitic masses (Lee & de Freitas, 1989)

2.2 Strength characteristic of the weathered rock

Already mentioned in Chapter 1, the weathered rock has intermediate characteristics between soil and rock. Strength of the weathered rock is one of them. Generally, the shear strength of the geo-material is expressed as effective cohesion and effective internal friction angle based on Mohr-Coulomb failure criteria shown below.

$$\tau'_{ff} = c' + \sigma'_n \tan \phi'$$

Where,

τ'_{ff}: shear strength of the material

c': effective cohesion of the material

σ'_n: effective confining stress

ϕ': effective internal friction angle

When considering idealized soil, Cohesionless soil literally has only internal friction angle without cohesion and conversely cohesive soil has only cohesion without internal friction angle. However, intact rock has both y-intercept and internal friction angle. The reason why using the term y-intercept not the cohesion is that it is irreplaceable bonding once the intact rock is broken unlike cohesive soil.

Present study performed full coring of the weathered rock in two boreholes. Recovering high quality weathered rock cores is very difficult because the weathered rock has somewhat low strength for bending or extending. Generally, drilling procedure highly damage the quality of cores. Therefore, To guarantee of quality of cores, many attempts were applied. A polymer, which reduces the friction between drilling bit and the ground, was applied. Length of the core reset 3 m, normal in companies, to 1 m. Triple core barrel was applied. The recovered high quality core was like figure 2.2.

Figure 2. 2 Recovered cores of weathered rock layer in present study

As you can see in the figure 2.2, the recovered core still has rock structure. Furthermore, UCS of the weathered rock core have reported and present research suggest UCS of the recovered core. Therefore, the weathered rock is similar with intact rock in the way that it has y-intercept.

Already mentioned in chapter 2.1.2, rock mass is considered as a discontinuum material. Then, the shear strength of the rock mass is reduced from the shear strength of the intact rock based on condition of discontinuity. However, when it comes to the weathered rock, assumption of discontinuum material is not proper because clear discontinuity does not appear in the core and even a part of the core seems to intact that the part has no sounding as a rock. Therefore, the weathered rock should be considered as a continuum material. This characteristic is similar with soil than rock.

2.3 Basic information of applied field test

PressureMeter Test (PMT) and Borehole Shear Test (BST) were performed in present study. Chapter 2.3 shows basic information related with PMT and BST such as an introduction, procedure, and interpretation methods for the tests.

(1) Introduction

(2) Procedure

(3) Calibration

(4) Interpretation methods

2.3.1 PressureMeter Test (PMT)

Introduction

Pressuremeter is a probe, which has expansional membrane and can take constant pressure to the cavity wall (Amar et al. 1991). There are many different types of PMT depending on the way to make a pressure to the cavity wall and to measure the deformation of the cavity. However, irrespective of the types of PMT, PMT is a test, which insert the probe into the cavity, make a pressure to the cavity wall and measure the deformation of it.

Present study used Elastmeter-2 made by OYO corporation in Japan which is a type of pressuremeter. To insert the probe into the cavity, borehole was made by drilling machine because the weathered rock layer is too hard to drill by pressuremeter itself. Elastmeter-2 measure the deformation of the cavity by the arm inside of the probe. It can measure the radius of the cavity wall.

Test Procedure

Test procedure of PMT is simple; (1) insert the probe into the target depth (2) pressure the cavity wall (3) measure the deformation of the cavity wall. However, details of the test are not unified. Here is the table, which summarize the details of the PMT along the several institutions.

Table 2. 2 Details of the PMT control methods (Clarke, 1995)

Proposer	Nation	Type of PMT	Ground type	Control method	Increment of applied pressure	Increment of displacement	Remark
LCPC	France	Menard (PBP)	Soil /Rock	Pressure	$\frac{P_{limit}}{10}$	-	
GOST	Russia	PBP	Soil	Pressure	0.25 MPa	-	
ASTM	U.S.A	PBP	Soil /Rock	Pressure /Displacement	0.25 ~2 MPa	0.05 ~ 0.1 V_o	Withdraw (2016)
Mair & wood (1987)	England	PBP, SBP	Soil /Rock	Pressure /Displacement	Soft clay < 15 kPa Stiff clay < 50 kPa Weak rock > 0.1 MPa	At least 6 points before yielding point	
OYO corporation	Japan	Elastmeter	Soil /Rock	Pressure	0.5 MPa	-	

Present study follows the basic procedure recommended by OYO Corporation. Additionally, to observe clear linear section and yielding point,

increment of pressurization was applied 0.1 MPa. After yielding point, increment of pressurization increased to 0.5 MPa and the test was performed until the deformation and pressure capacity of the device.

Calibration

Calibration for PMT is different from the manufacturer because the device is different. Therefore, present study refer the manual from the OYO Corporation. There are three types of calibration steps for Elastmeter-2 used in this study; (1) zero set calibration between the arm and data logger (2) compression of rubber tube caused by the applied pressure (3) thinning of the rubber tube caused by the expansion of itself.

(1) Zero set calibration between the arm and data logger

Elastmeter-2 have a measuring system, which read the displacement of the arm inside of the probe. Therefore, it need to be zero set correspondingly. For the calibration, calibration ring is applied to the probe without the rubber tube on the two points, (a) and (b), as shown in figure 2.3.

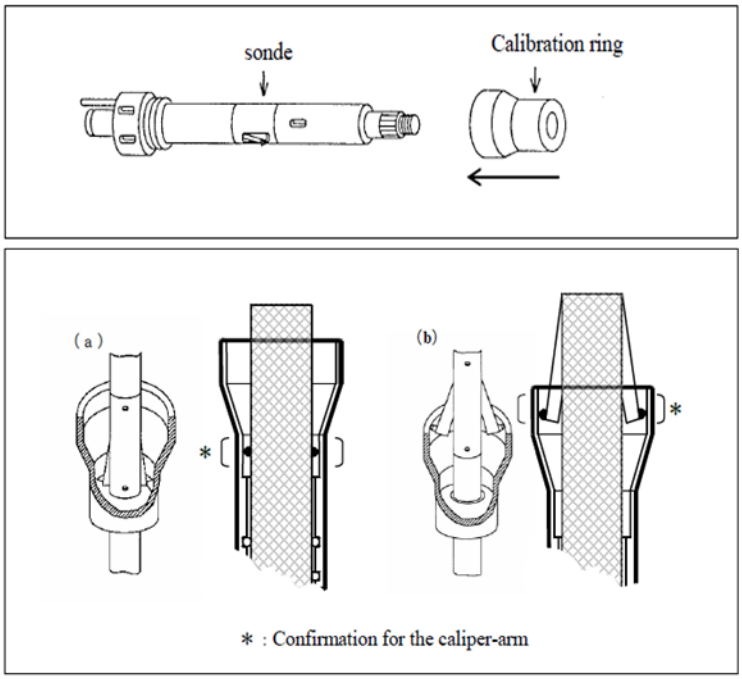


Figure 2. 3 Schema of combining procedure between probe and calibration ring (OYO, 2008)

The arm need to be set when calibration ring is applied on (a), data logger should show -0.02 ~ 0.02 mm, and being applied on (b), it should show 9.98 ~ 10.02 mm.

(2) Compression of rubber tube caused by the applied pressure

The radius which shown on the data logger is a radius of the cavity wall with rubber tube considered as constantly 23.5 mm. Therefore, the compression of the rubber tube caused by the applied pressure overestimate the actual radius of the cavity wall.

For this calibration, the probe with rubber tube is inserted into the

calibration pipe centralized with guide ring. After applied pressure is larger than 1 MPa, the probe abuts on the calibration pipe. The applied pressure and the read displacement organized like table 2.3.

Table 2. 3 Measurement data sheet concerned with the thickness variation volume (OYO, 2008)

Pressure (P) (MPa)	**Displacement (R_p) (mm)**	**Pressure variation ($P' = P - 1$)**	**Thickness variation volume ($R' = R_p - R_{p_1}$)**
1.0		0.0	
1.5		0.5	
:		:	
20.0		19.0	

Then, the inclination k is calculated with the graph plotted on applied pressure versus displacement like figure 2.4.

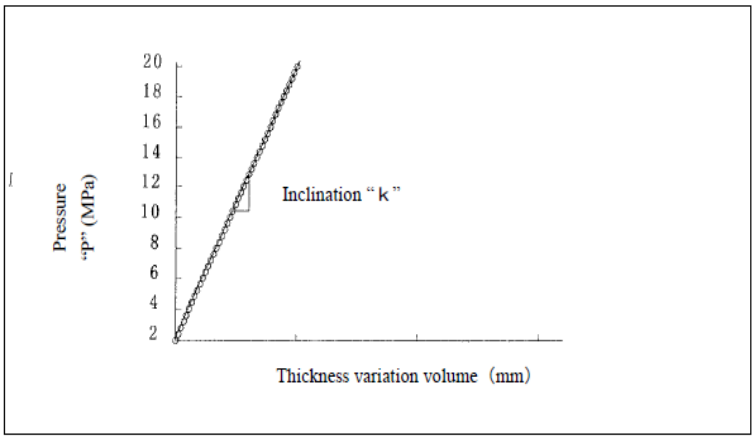


Figure 2. 4 Thickness variation volume responding to the respective pressure account for the corrected values (OYO, 2008)

Calibration for compression of the rubber tube is not needed under applied pressure 1 MPa. After pressure range of 1 MPa, calibration is performed following these formula.

$R_s = R_i - PG$ (mm)

$PG = \frac{(P-1)}{K}$ (mm)

R_s : inside radius of the rubber tube

R_i : read data

PG : calibration coefficient of compression of rubber tube

K : inclination on the graph plotted on pressure versus displacement

(3) thinning of the rubber tube caused by the expansion of itself

During the expansion, rubber tube experience thinning effect in the radial

direction. Calibration for thinning effect is performed under the assumption that the area of the rubber tube is not change during the expansion like figure 2.5. The area of the rubber tube can be calculated on the calibration pipe test performed on the calibration for compression of rubber tube shown below.

$$S = (R_{pipe}^2 - \left(R_{p1} + 23.5\right)^2 \times \pi$$

R_{p1}: read displacement corresponding with applied pressure 1 MPa

R_{pipe}: inner radius of calibration pipe

23.5 mm: initial thickness of rubber tube

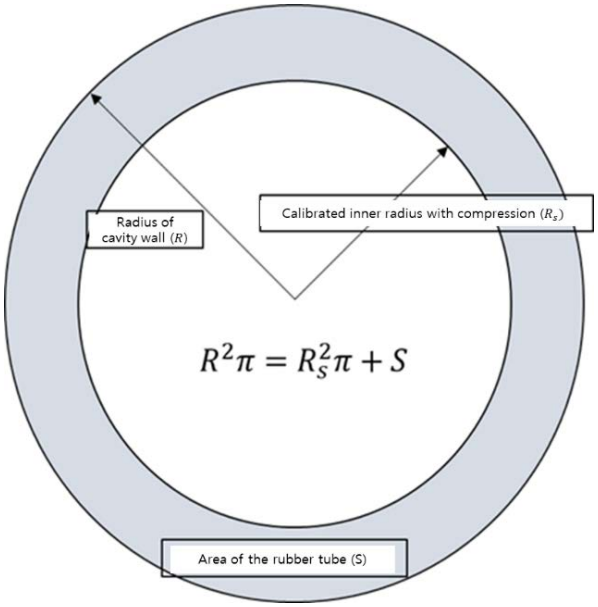


Figure 2. 5 Conceptual schema for the calibration of thinning effect

Interpretation methods

The basic theory for the PMT is cavity expansion theory. The cavity expansion theory is about the stress-strain relationship under condition that radial stress applied to the cylindrical cavity with the infinite length of vertical way. The basic assumptions for cavity expansion theory are as follows.

(a) Adjacent ground is not disturbed during the formation of the cavity.
(b) Ground is homogenous and isotropic.
(c) Cavity is vertical with the ground surface and the ratio between vertical length and radius of the cavity is infinite.
(d) Cavity is under axisymmetric condition.

A situation for cavity expansion with the applied pressure can be described as shown in figure 2.6.

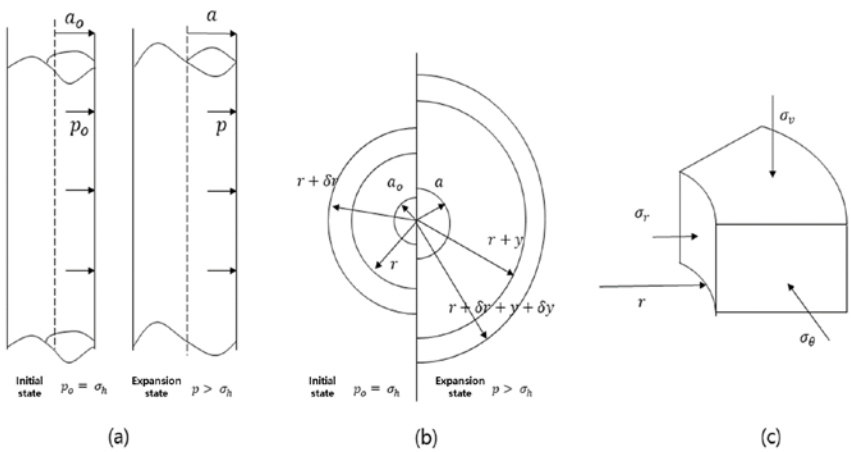


Figure 2. 6 The definitions used in the analysis of the expansion of a cylindrical cavity: (a) expansion of a cylindrical cavity; (b) expansion of an element at radius *r*; (c) stresses on an element at radius *r* (Clarke, 1995)

Meaning of the terms used in figure as follows. a_o, a are the radius of the cavity and subscription o means initial condition. p_o, p are the applied pressure in the equilibrium state. σ_h is an in-situ horizontal earth pressure. In figure (b), here is a ground element with a thickness δr located a distance of r from the center of the cavity. The inner radius of the element expands to $r + y$ and the thickness to $\delta r + \delta y$ as the applied pressure is increased. In figure (c), there are three principal stresses; vertical σ_v, radial σ_r, and circumferential σ_θ, on the element at radius r.

Timoshenko and Goodier (1934) show that the equation of equilibrium is

$$\frac{d\sigma_r}{dr} = -\frac{\sigma_r - \sigma_\theta}{r}$$

Thus, the tensile circumferential strain, ϵ_θ, is

$$\epsilon_\theta = \frac{y}{r}$$

Since the circumference increases from $2\pi r$ to $2\pi(r+y)$.

The thickness of the element changes by δy, therefore the radial strain, ϵ_r, is

$$\epsilon_r = \frac{\delta y}{\delta r}$$

The only variables measured in PMT are the applied pressure, p, and the radius of the cavity wall, a. The circumferential strain at the cavity wall is referred to as the cavity strain, ϵ_c, which is defined as

$$\epsilon_c = \frac{a - a_o}{a_o}$$

It is assumed that at some distance from the probe the strain and the change in radial stress are zero; that is, $\sigma_r = \sigma_\theta = \sigma_v$ and $\delta y = 0$.

Consider a cavity expanded in an ideal linear elastic soil. The principal strains are related to the principal stresses change by

$$E\epsilon_r = \Delta\sigma_r - \nu(\Delta\sigma_\theta + \Delta\sigma_v)$$

$$E\epsilon_\theta = \Delta\sigma_\theta - \nu(\Delta\sigma_v + \Delta\sigma_r)$$

$$E\epsilon_v = \Delta\sigma_v - \nu(\Delta\sigma_r + \Delta\sigma_\theta)$$

Where E is the modulus of elasticity and ν is Poisson's ratio. The vertical strain is zero because the cavity has infinite length of vertical section. Therefore,

$$\Delta\sigma_v = \nu(\Delta\sigma_\theta + \Delta\sigma_r)$$

Combining equations gives the differential equation

$$r^2\frac{d^2y}{dr^2} + r\frac{dy}{dr} - y = 0$$

The boundary conditions are

$$y = 0 \text{ for } r = infinity$$

$$y = (a - a_o) \text{ for } r = a$$

The displacements and stresses within the soil are given by

$$y = \epsilon_c \frac{a_o a}{r}$$

$$\Delta\sigma_r = \sigma_r - \sigma_h = 2G\epsilon_c \frac{a_o a}{r^2}$$

$$\Delta\sigma_\theta = \sigma_\theta - \sigma_h = -2G\epsilon_c \frac{a_o a}{r^2}$$

Where G is the shear modulus and σ_h is the total horizontal stress. The radial and circumferential strains at distance r are equal and opposite and, since there is no vertical strain, there must be no volume changes in the ground.

At the cavity wall, $r = a$, and $\sigma_r = p$. During the initial loading as $a - a_o$ is small, equation can be written as

$$p - \sigma_h = 2G\epsilon_c$$

Hence, for a linear elastic zone, the shear modulus of the ground is given, in terms of cavity strain, as

$$G = \frac{0.5(p - \sigma_h)}{\epsilon_c}$$

After the ground get out from the elastic zone, behavior of the ground is influenced by drainage condition. The weathered rock layer can be assumed as drained condition. Under the drained condition, the behavior of the ground is affected by the volumetric change. There have been lots of previous studies performed to get a stress-strain relationship after the yield point. Ladanyi (1963) suggested failure happens under constant stress ratio and constant

volume. Vesic (1972) suggested the general solution for the ground, which have both effective cohesion and effective internal friction angle, using limit pressure, p_l and rigidity factor, I_{rr}'. Sayed (1989) suggested a method for calculating volumetric strain by performing several lab tests with sand. However, previous studies have limitation that many triaxial compression tests should be performed to determine volumetric strain at failure.

To overcome this limitation, Hughes et al. (1977) suggested a method to determine maximum internal friction angle for very dense sand assuming constant ratio of volumetric strain. Robertson and Hughes (1986) advanced previous study to several types of sandy soil. Haberfield and Johnston (1990) suggested a method for calculating strength parameters of weak rock layer from PMT with theoretical approach.

2.3.2 Borehole Shear Test (BST)

Introduction

Borehole Shear Test, BST is a direct shear test performed at the field. Test result of BST is same as direct shear test performed in laboratory. *The concept of Borehole shear, determining shear strength parameters on the sides of a test borehole, presents a rather unique solution to the problem of in situ evaluation of shear strength of soil and rock* (Lutenegger and Hallberg, 1981). BST is generally applied to weathered rock layer to determine strength parameters of it in Korea.

Test Procedure

Test procedure of BST is simple; (1) insert the shear plate into the target depth (2) pressure the cavity wall to the target normal stress (3) pull the shear plate with measuring shear stress.

Present study used borehole shear device made by Handy Geotechnical Instruments. The BST devices consist of shear plate, pulling device, pressure device, and shear stress measuring device.

Area of shear plate is 32.3 cm^2. Shear plate have a teeth to make soil-to-soil shearing, rather than soil-to-plate. Teeth are composed of twenty-five 60 deg wedges, spaced at 2.5 mm, as shown in figure 2.7.

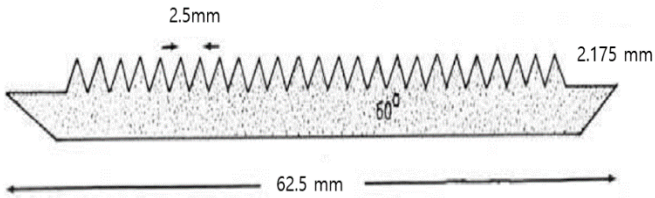


Figure 2. 7 Shear plate design (Lutenegger & Hallberg, 1981)

Table 2.4 summarizes applied test condition.

Table 2. 4 Applied test conditions

Test condition	Values
Applied normal stress (kPa)	50, 100, 150, 200
Stabilizing time for normal stress (min)	5
Velocity of shearing (mm/s)	0.04
Final displacement of shear plate (mm)	4

Handy and Fox (1967) suggested sequential staged test method which is successive shears at progressively higher normal pressures with the shear plate is left in place. Present study adopted sequential staged method for the effective test procedure.

Stabilizing time and velocity of shearing are the concepts related to drainage condition. Handy et al. (1985) suggested that stabilizing time is 5 to 10 min. ASTM D3080 (2011) suggested 0.03 mm/s of shearing velocity for the direct shear test in the laboratory. Present study applied 5 min for stabilizing time and 0.04 mm/s for shearing velocity because the weathered rock layer has good drainage condition.

Calibration

Gauge for shear stress measure the load, which makes shear plate lower in the opposite direction of plate movement. Therefore, the gauge can display

certain value before the shearing. That value means the dead load of the rod used to insert the plate into the desired depth. Therefore, it need to be zero set.

Interpretation methods

BST test data is the sets of maximum shear stress under the certain normal stress. Each data point is plotted on the plane of normal and shear stress. Then, cohesion and internal friction angle can be obtained.

Chapter 3 Experimental Result

3.1 Introduction

Tests were performed at Goyang city. Three boreholes were drilled. The first one was to verify ground layers with standard penetration test prior to the main tests. Second was to recover the samples and cores. The last one was to perform the tests, PMT and BST.

3.2 Boring log and recovered cores

Figure 3.1 shows the profile of the test ground, which consists of reclamation soil, deposit soil, residual soil, weathered rock, soft rock layers and ground water table, 5.0 m. From ground surface to the depth of 3.5 m, there was reclamation soil layer, which was classified as SM in USCS (Unified Soil Classification System). Modified SPT blow count N_{60} of this layer 10 ~ 23. Below the reclamation soil layer, to the depth of 25.0 m, there was two different types of deposit soil layers, upper one was classified as ML and CL in USCS and lower one was classified as GM in USCS. N_{60} of upper deposit layer was 3 ~ 9, and lower one was 33 ~ 115. Lower deposit soil layer was gravels located on the riverbed. After deposit layers, there was a thin residual soil layer, which was classified as SM in USCS with the thickness of 1 m. Beneath the residual soil layer, a weathered rock (granite) layer existed to the depth of 39.0 m. N_{60} of the weathered rock layer recorded only

5 times because it showed larger than 300 continuously on 3 times. At the bottom of the ground, there was soft rock (granite) layer, whose rock quality designation (RQD) was 30 ~ 50.

Depth (m)	Thickness (m)	Layer	USCS	Rock quality		Discontinuity			SPT	
				TCR (%)	RQD (%)	Max (mm)	Min (mm)	Ave (mm)	Blows	Embedment (cm)
1.0	3.5	Reclamation soil layer	SM						23	30
2.0									10	30
3.0									10	30
4.0	21.5	Deposit soil layer	ML or CL						7	30
5.0									3	30
6.0									9	30
7.0									5	30
8.0									4	30
9.0									6	30
10.0									5	30
11.0									5	30
12.0									6	30
13.0									5	30
14.0									11	30
15.0			SM or GM						15	30
16.0									50	22
17.0									50	18
18.0									33	30
19.0									50	15
20.0									50	26
21.0									50	24
22.0									50	16
23.0									50	14
24.0									50	13
25.0	1.0	Residual soil layer	SM						50	14
26.0									50	10
27.0	13.0	Weathered rock layer	WR						50	6
28.0									50	3
29.0									50	5
30.0									50	4
31.0										
32.0										
33.0										
34.0										
35.0										
36.0										
37.0										
38.0										
39.0										
40.0	4.5	Soft rock layer	SR	100	40	16	1	8		
41.0				100	50	18	1	10		
42.0										
43.0				100	30	14	1	7		
44.0	End of boring									

Figure 3. 1 Boring log in present study

For the observation and the lab test, cores of the weathered rock were recovered very carefully. To enhance the quality and recovery of the cores, intervals of the drilling was set as 1 m rather than 3 m, which is practically

applied, in addition, polymer was inserted into the boring water, which reduces friction between drilling bit and the ground. Figure 2.2 shows the cores recovered from the borehole.

3.3 Result of the tests

3.3.1 PMT

PMT was performed total 10 times with interval of 1.2 m along the depth. The test was conducted once in residual soil layer, twice in the soft rock layer, and other seventh in the weathered rock layer.

Applied pressure and the radius of the cavity wall can be obtained with data calibration, which is mentioned in Chapter 2. Figure 3.2 shows the representative test result with applied pressure and radius of cavity wall. In this figure, effective horizontal pressure, p_o, and effective yield pressure, p_y can be determined. p_o is the pressure which is the starting point of linear portion of the curve, and p_y is the pressure which is the end point of the linear portion of the curve (Kim et al., 2000). Once p_o, p_y are determined, deformation modulus E_m can be calculated. E_m is called Menard modulus or pressuremeter modulus, an initial elastic modulus taken from the slope between two points corresponding with p_o, p_y (Clarke, 1995; OYO, 2008). Table 3.1 summarize test results performed in present study.

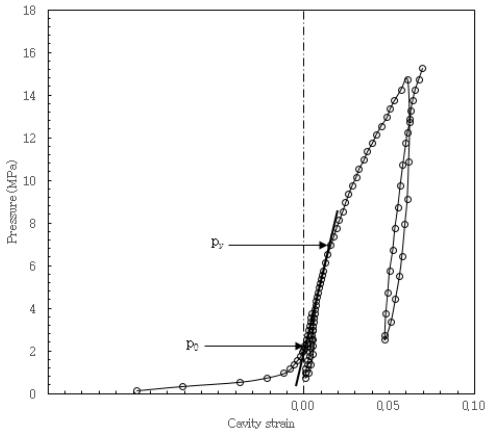


Figure 3. 2 representative PMT curve at the depth of 31.6 m

Table 3. 1 PMT results along the depth performed in present study

Depth (m)	p_o (MPa)	p_y (MPa)	E_m (MPa)
26.80	0.99	3.26	360.88
28.00	1.87	7.79	521.15
29.20	1.56	5.56	460.00
30.40	1.15	6.59	532.31
31.60	2.04	6.63	442.28
32.80	1.13	3.31	242.05
34.00	1.52	5.63	401.24
35.20	1.10	6.16	2305.23
36.40	2.29	9.66	1576.34
39.50	2.46	7.92	886.55
41.00	1.65	10.34	4250.35

As already mentioned before in Chapter 2, Haberfield and Johnston (1990) suggested a method for the determining strength parameters for the material, which has both cohesion and internal friction angle. This method determine the strength parameters of the material comparing the curve of applied pressure and radius of cavity wall from PMT after yielding point with theoretical curve. Haberfield (1987) suggested theoretical curve of applied pressure and the radius of cavity wall from PMT as shown below

Assumptions

(1) Ground follows Mohr-Coulomb failure criteria

(2) Ground follows dilatancy theory (Hughes et al. 1977)

$$\sin\psi' = \frac{\sin\phi' - \sin\phi'_{cv}}{1 - \sin\phi' \sin\phi'_{cv}}$$

Where,

ϕ': effective internal friction angle

ϕ'_{cv}: effective residual friction angle

ψ': dilatacy angle

(3) Effect of radial crack does not be considered

Formula

$$p'_o \leq p' \leq p_y{}',\ \frac{a-a_o}{a_o} = \frac{p'-p'_o}{2G}$$

$$p'_y \leq p',\ \frac{a-a_o}{a_o} = \frac{1}{2G}\left[b_1\left(\frac{a_y}{a_o}\right)^{\frac{m-1}{m}} + b_2\left(\frac{a_y}{a_o}\right)^{\frac{n+1}{n}} + b_3\right]$$

$$b_1 = \frac{-2m}{m-1}\left[(1-\nu')\left(\frac{1+mn}{m+n}\right) - \nu'\right](p'_y - p'_o)$$

$$b_2 = 2n(1-\nu')\left(\frac{m+1}{m+n}\right)(p'_y - p'_o)$$

$$b_3 = (1-2\nu')\left(\frac{m+1}{m-1}\right)(p'_y - p'_o)$$

$$\frac{a_y}{a_o} = \left[\frac{p'(m-1)+\sigma^*}{p'_y(m-1)+\sigma^*}\right]^{\frac{m}{m-1}}$$

$$\sigma^* = \frac{2c'\cos\phi'}{1-\sin\phi'}$$

$$m = \frac{1 + sin\,\phi'}{1 - sin\,\phi'}, n = \frac{1 + sin\,\psi'}{1 - sin\,\psi'}$$

Where,

p': effective pressure applied to cavity wall (p'_o: initial, p'_y: yield)

a: radius of cavity wall (a_o: initial, a_y:yield)

G: shear moduli of the ground

ν': Poisson's ratio

c': effective cohesion

To draw the curve of applied pressure and radius of cavity wall following above formulas, six parameters, $c', \phi', \psi', G, p'_o, \nu'$ should be determined. G and p'_o can be determined by the curve from PMT. ν' is recommended to use reference value because it has small range comparing with other parameters. c', ϕ', ψ' cannot be determined directly but there is two formula shown below which can describe relationship between above three parameters.

$$p'_y = p'_o(1 + sin\,\phi') + c'\,cos\,\phi'$$

$$\sin\psi' = \frac{\sin\phi' - \sin\phi'_{cv}}{1 - \sin\phi'\sin\phi'_{cv}}$$

ϕ'_{cv} can be determined by lab test or reference value because it is relatively small range same as ν'. Then, to determine remained three parameters, $c', \phi', p_y{}'$, trial $\phi', p_y{}'$ applied repetitively. Trial ϕ' is ranged

from ϕ'_{cv} to 90° and trial $p_y{}'$ is ranged from 0.8 times to 1.2 times of $p_y{}'$ determined by PMT. Therefore, final defined values of strength parameters c', ϕ' can be determined which shows the smallest deviation comparing with actual PMT.

Table 3.2 summarize strength parameters determined by Haberfield and Johnston method.

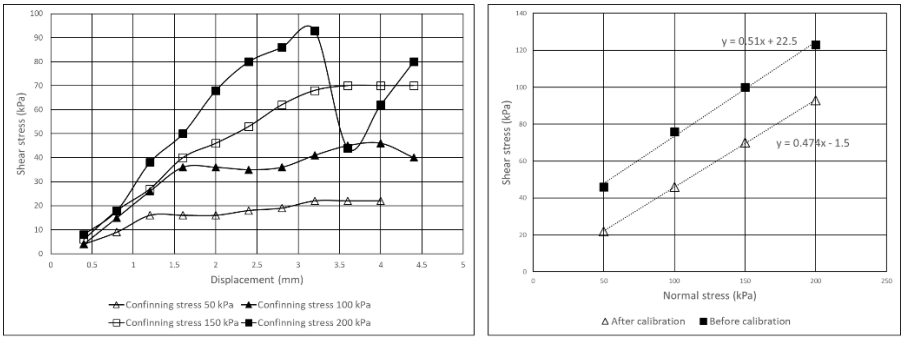
Table 3. 2 Shear strength parameters determined by Haberfield and Johnston method in present study

Depth (m)	c' (MPa)	ϕ' (°)	Error (%)
26.80	1.61	43.00	0.65
28.00	5.34	49.50	2.60
29.20	4.31	42.50	0.95
30.40	4.95	40.50	2.19
31.60	2.68	50.00	1.54
32.80	1.27	43.50	0.95
34.00	2.79	47.00	0.85
35.20	4.12	40.50	32.47
36.40	9.00	40.50	4.27
39.50	3.63	41.00	1.04
41.00	7.34	40.50	11.45

3.3.2 BST

Test results of direct shear test in lab can be deduced the curves, which are load-horizontal displacement curve, volumetric change-horizontal displacement curve, and shear stress-normal stress curve (Bowles, 1992). Same as direct shear test in lab, test results of BST can be deduced the curves, which are shear stress-displacement and shear stress-normal stress curve. However, volumetric change cannot be deduced in BST because radial displacement is not measured. In a strict way, displacement of the plate is not same as the displacement of ground. So shear stress-displacement curve from BST could be used only classifying the behavior of the ground such as hardening and softening.

Present study performed BST total seven times with the depth, which conducted mainly on the weathered rock layer for six times, only a case on the residual soil layer. Figure 3.3 shows the representative shear stress-plate displacement curve and shear stress-normal stress curve which conducted in the depth of 30.6 m. Table 3.3 shows the whole test results with the depth both before and after the calibration.



(a) (b)

Figure 3. 3 Representative test result of BST at the depth of 30.6 m: (a) Shear stress-plate displacement curve; (b) Shear stress-normal stress curve

Table 3. 3 Test results from BST in present study

Depth (m)	Before calibration		After calibration		Layer
	c' (kPa)	ϕ' (°)	c' (kPa)	ϕ' (°)	
25.50	36.93	19.14	-0.11	19.61	Residual Soil
26.80	15.50	22.49	-2.50	20.61	Weathered Rock
27.70	20.50	17.54	-0.50	15.43	
30.60	22.50	27.02	-1.50	25.36	
31.80	58.00	6.84	24.00	11.31	Strange value
34.00	12.00	29.94	-11.50	27.83	Weathered rock
35.70	15.50	30.88	-12.50	29.25	

3.3.3 Laboratory tests

To verify field test results, laboratory tests were performed which are six times of unconfined compression test and four times of triaxial compression test with measuring of wave velocity of cores. As mentioned before in chapter 3.2, it needs much carefulness to recover the high quality cores in weathered rock layer.

Table 3.4 and 3.5 show the test results of unconfined compression and triaxial compression test.

Table 3. 4 Test results of unconfined compression

Unconfined Compression Test					
Depth (m)	**Size of specimen $D \times L$ (mm)**	**Wave velocity (m/sec)**		**Compressive strength (MPa)**	**Layer**
		P wave	**S wave**		
28.0 – 28.1	50.3 × 150.35	831	562	4.4	Weathered Rock
31.7 – 31.8	50.3 × 82.60	798	515	5.0	
32.2 – 32.3	50.3 × 126.35	592	381	1.6	
33.5 – 33.6	50.3 × 120.45	860	519	4.6	
40.7 – 40.8	50.6 × 105.10	5005	2661	192.7	Soft rock
42.2 – 42.3	50.6 × 109.35	5816	3170	202.7	

Table 3. 5 Test results of triaxial compression

Triaxial Compression Test							
Sample number	**Depth (m)**	**Size of specimen $D \times L$ (mm)**	**σ_3' (MPa)**	**σ_1' (MPa)**	**c' (MPa)**	**ϕ' (°)**	**Layer**
1-1	29.0 – 29.1	50.3 × 111.60	0.5	11	1	61	Weathered rock
1-2	30.0 – 30.1	50.3 × 124.00	1	19			
2-1	31.6 – 31.7	50.3 × 104.55	0.5	14	1	61	
2-2	31.5 – 31.6	50.3 × 109.25	1	22			
2-3	31.6 – 31.7	50.3 × 81.10	1.5	28			
3-1	33.0 – 33.1	50.3 × 97.55	0.5	14	1	61	
3-2	33.3 – 33.4	50.3 × 123.80	1	20			
3-3	34.5 – 34.6	50.3 × 107.25	1.5	27			
4-1	40.8 – 40.9	50.6 × 109.50	5	255	26	59	Soft rock
4-2	41.3 – 41.4	50.6 × 111.95	10	336			
4-3	42.1 – 42.2	50.6 × 109.45	15	389			

Chapter 4 Discussion

4.1 introduction

The experimental results of shear strength parameters of the weathered rock layer are summarized below table 4.1.

Table 4. 1 Experimentally estimated shear strength parameters in present study

Depth (m)	PMT		BST		Lab Triaxial		Layer
	c' (MPa)	ϕ' (°)	c' (MPa)	ϕ' (°)	c' (MPa)	ϕ' (°)	
25.5			0	19.61			Residual soil
26.8	1.61	43.0	-0.003	20.61			Weathered Rock
27.7 ~ 28.0	5.34	49.5	-0.001	15.43			
29.0 ~ 30.6	4.31	42.5	-0.002	25.36	1	61	
31.5 ~ 31.8	2.68	50.0	0.024	11.31	1	61	
32.8 ~ 34.6	1.27	43.5	-0.012	27.83	1	61	
	2.79	47.0					
35.2 ~ 35.7	4.12	40.5	-0.013	29.25			
36.4	9.0	40.5					
39.5	3.63	41.0					Soft rock
40.8 ~ 42.2	7.34	40.5			26	59	

The unconfined compressive strength (UCS) of the weathered rock layer determined by the lab tests is ranged from 1.6 to 5.0 MPa. It is well matched result with classification system for the weathered rock layer previously mentioned in chapter 2; 0.2 ~ 60 MPa (Korea Expressway Corporation, 2009), lower than 5 MPa (Korea Rail Network Authority, 2011), and lower than 10 MPa (Seoul Metropolitan Government, 2006). Moreover, the soft rock layer on present study shows much higher UCS values; 192.7, 202.7 MPa and cohesion; 26 MPa than those of the weathered rock layer. Therefore, the ground classification applied on present study is reasonable.

The shear strength parameters of the weathered rock layer are determined from the two field tests, PMT and BST, and the lab triaxial compression test. Three tests show considerably different results about both strength parameters, cohesion and internal friction angle. Cohesion shows negative values from the BST, 1 MPa from the lab triaxial compression test, which shows same value on three different cores of the weathered rock with depth, and 1.27 ~ 9.00 MPa from the PMT. Internal friction angle shows 15 ~ 29° from the BST, 40 ~ 50° from the PMT, 61° from the lab triaxial compression test.

4.2 Laboratory tests

Present study performed two tests, unconfined compression test and triaxial compression test on the recovered cores. Although present study

recover the cores with much care, figure shows a difference of wave velocity between in field and in core.

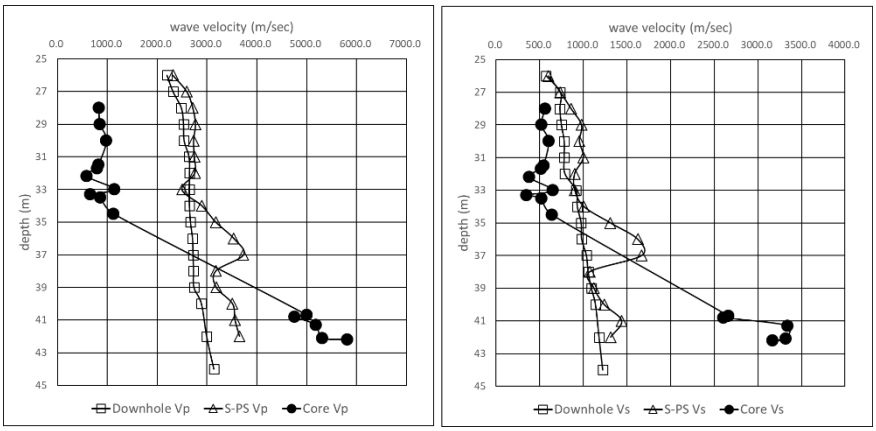


Figure 4. 1 Wave velocity of field ground and of core

Wave velocity on the weathered rock shows higher value in field than in core and contrastively the wave velocity on the soft rock shows higher value in core than in field. Decreasing of the wave velocity between in field and in core on the soft rock is because the soft rock is a discontinuous material, which has many discontinuities. However, in the case of the weathered rock, difference of wave velocity implies that the weathered rock core disturbed during and after the coring. Therefore, the test result from the lab tests can underestimate the shear strength parameters of the weathered rock.

4.3 PMT

Present study try calculating shear strength parameters by Haberfield and Johnston method from PMT result. However, as summarized in chapter 4.1, estimated shear strength is different with the result from lab test. There are three possible facts to affect the final estimation.

(1) Estimation of p_o'

(2) Selection of G

(3) Determination of ϕ_{cv}'

Estimation of p_o'

Present study applied pre-drilling method to make a borehole for PMT. It is the only option for the weathered rock layer because of the strength and stiffness of the weathered rock layer. However, pre-drilling causes inevitable unloading to the cavity wall. Therefore p_o', the pressure where the slope of the curve becomes linear, is not equal to $\sigma_h{}'$, effective horizontal stress of the ground. $\sigma_h{}'$ is somewhat larger than p_o' and it is very difficult to determine $\sigma_h{}'$ from pre-bored type PMT (Clarke, 1995). Haberfield and Johnston (1990) showed effect of p_o' to estimation of shear strength parameter shown in figure 4.2 by performing a test on typical rock properties.

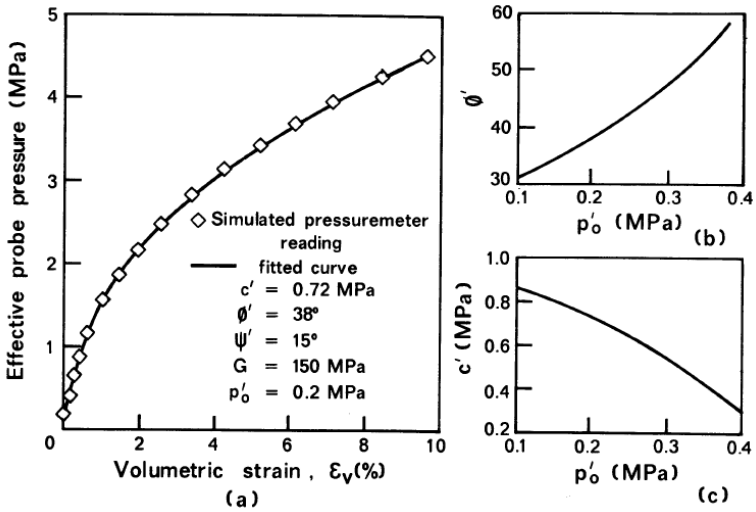


Figure 4. 2 A pressuremeter curve and the corresponding possible values of c', ϕ' and p'_o (Haberfield and Johnston, 1990)

Selection of G

Several previous studies recommended usage of G_{ur}, shear modulus of loading-unloading curve, rather than G_i, initial shear modulus which almost same as E_M, pressuremeter modulus, in the selection of G from PMT. G_{ur} depends on the range of shear strain. However, independent of the range, G_{ur} shows larger value than G_i.

Application of G_{ur} on Haberfield and Johnston method is impossible because of discordance of the yielding point between theoretical curve and actual pressuremeter curve. Present study adopts E_M for the stiffness of the ground to estimate the shear strength parameters. Figure 4.3 shows the case of applying larger G.

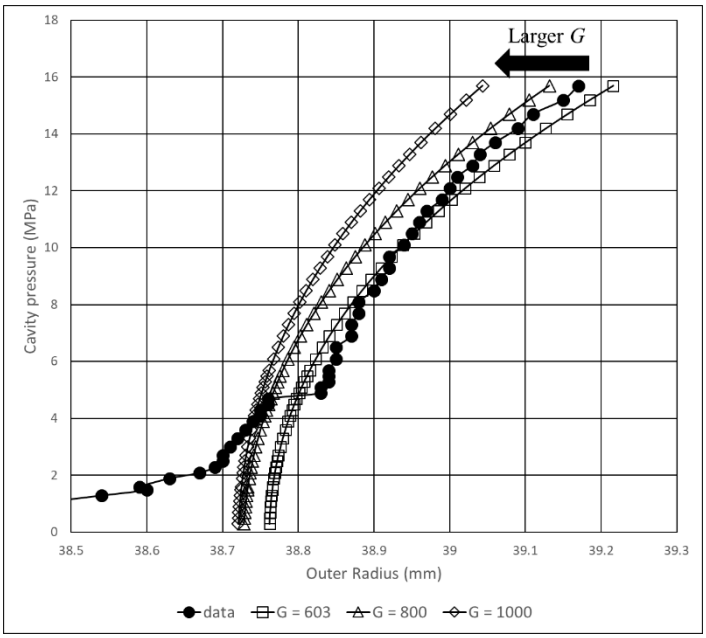


Figure 4. 3 Change of estimated PMT curves with larger *G*

Determination of ϕ'_{cv}

Haberfield and Johnston (1990) recommended usage of reference value for ϕ'_{cv} because it has small range comparing with other factors. However, there is not many suggested values for weathered granite. Table 4.2 shows change of suggested ϕ'_{cv} as weathering.

Table 4. 2 Proposed ϕ'_{cv} by previous researches

Proposer	Classification		ϕ'_{cv}
Lee & Chang (2003)	Absorption (%)	9.5	41.3
		10.1	33.2
Robertson & Hughes (1985)	Soil type	Well-graded Gravel-sand-silt	40
		Uniform coarse sand	37
		Well-graded medium sand	37
		Uniform medium sand	34
		Well-graded fine sand	34
		Uniform fine sand	30

Therefore, present study performed direct shear tests to determine ϕ'_{cv} of the weathered rock layer. Samples used in direct shear tests are two types. One is residual soil and the other is weathered rock. The sample of weathered rock is fully broken into particles to remove its rock structure. Figure 4.4 shows the results of direct shear test. ϕ'_{cv} in both residual soil and weathered rock are almost same; 39.11°, 39.79°.

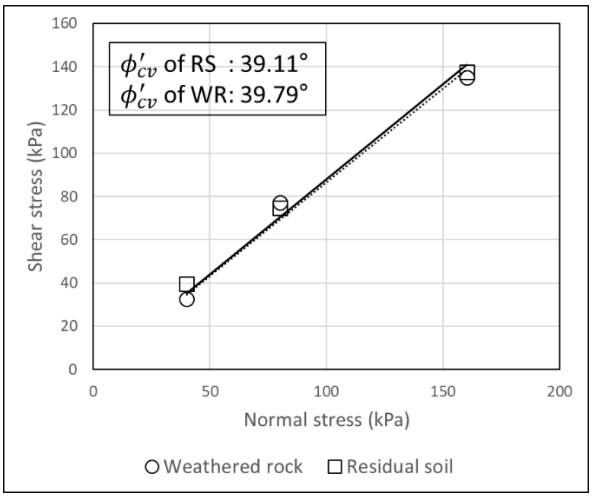


Figure 4. 4 ϕ'_{cv} from direct shear test

Therefore, present study applies 40° for ϕ'_{cv} on the residual soil, weathered rock, and soft rock layer.

4.4 BST

The cohesion estimated by BST appears negative values. Especially, cohesion estimated on the depth of 34.0 and 35.7 m shows larger than -10 kPa. Minimum value of cohesion or y-intercept in shear and normal stress plane is zero. Therefore, BST test result estimated in present study is not reliable. Handy et al. (1985) said that cohesion estimated by BST could be shown in negative value correspondingly with near 45° of internal friction angle like in figure 4.5 (c).

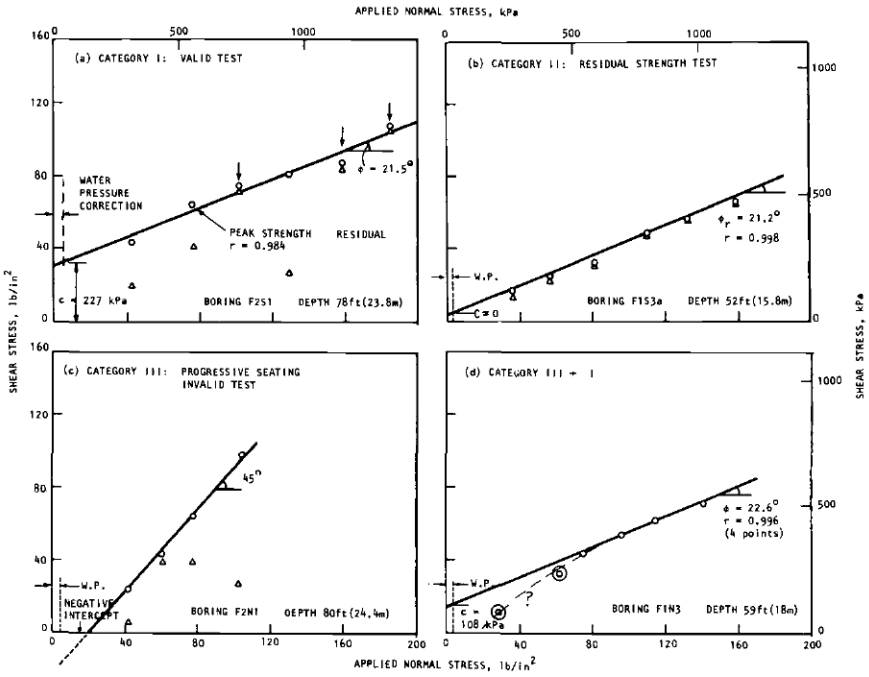


Figure 4. 5 Examples of different categories of state BSTs (Handy et al., 1985)

However, in present case, internal friction angle shows 15 ~ 29° much lower than 45°. Simon et al. (1990) reported similar case with present study. They performed BST for 63 times on saprolite in the depth of 0.2 ~ 3.0 m at Caribbean National Forest. They reported negative cohesion shown in half of the tests and correspondingly average internal friction angle was 28° lower than 45° shown in figure 4.6.

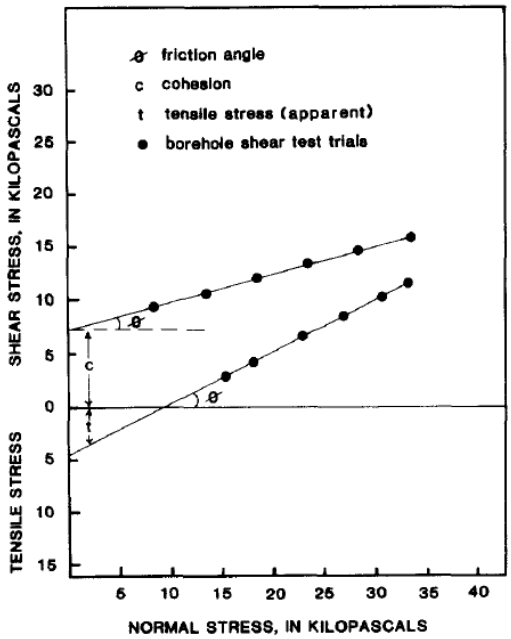


Figure 4. 6 Representative Mohr-Coulomb failure envelopes showing derivation of shear-strength parameters and a negative y-intercept, interpreted as tensile stress (Simon et al, 1990)

They suggested the reason of the negative cohesion is tensile stress inside of the saporite based on the observation of tensile crack on the head scarps.

Table 4.3 summarized previous result from BST on the weathered zone, which is weathered rock and residual soil. All data show much smaller value comparing with the result of lab test in present study. In addition, they are similar with the result of BST before calibration in present study.

Table 4. 3 Shear strength parameters of weathered zone from BST suggested by previous researches

Author	c' (kPa)	ϕ' (°)
Woo et al. (2010)	15.5	28.37
	25.0	30.75
	27.0	32.62
Song et al. (2013)	16.1	27.9
Kong et al. (2010)	36	33.8
	24	31.8

To apply BST on the weathered rock layer properly, there are some tasks to be improved. It should be checked that the shearing condition of boundary is properly set. Teeth of shear plate must penetrate properly into the ground. To verify the penetration of the teeth, suitable normal stress should be applied. Too weak normal stress can cause to shear between shear plate and the ground not between grounds themselves. Too large normal stress can cause not only shear between the grounds themselves but also passive resistance of the plate embedded into the ground.

Chapter 5 Conclusions

Field tests, PMT and BST and lab tests, unconfined compression test, triaxial compression test, shear wave velocity test are performed to estimate the shear strength of the weathered rock. In addition, applicability of the tests is evaluated by comparing with the tests result. Based on the results and analysis, the following conclusion is determined:

1. The weathered rock is a geo-material which has both cohesion and internal friction angle. Cohesion of the weathered rock is irreplaceable unlike clay soil because it comes from the structure of rock material. The fact that the weathered rock has both strength parameters makes interpretation of tests result difficult.

2. Pressuremeter test (PMT) is performed for ten times along the depth. Test result of PMT is shown as the curve of applied pressure and displacement of cavity wall. To determine shear strength parameters from the PMT curve, Haberfield and Johnston (1990) method is applied. Table 3.2 shows estimated cohesion and internal friction angle.

3. Borehole shear test (BST) is performed for seven times along the depth. From the BST, maximum shear stress is obtained with each normal stress. Shear strength parameter is directly calculated from the tests with different normal stress condition. Table 3.3 shows estimated cohesion and internal friction angle.

4. Lab tests, unconfined compression test and triaxial compression test with measurement of wave velocity test are performed for ten times totally. Table 3.4 and 3.5 show the lab test results with the size of specimen and recovered depth.

5. The whole test results from PMT, BST, lab tests, and previous research are compared. (1) Estimated strength parameters from PMT shows larger cohesion and smaller internal friction angle than lab tests. The reason of these results is the effect of input parameters used in Haberfield and Johnston method. (2) Estimated strength parameters from BST shows smaller cohesion and internal friction angle than lab tests. Estimated cohesion from BST shows negative value. Therefore, BST is not applicable on the weathered rock layer with current state. (3) Lab test results are considered as a true value for comparison of each results. However, wave velocity in the cores shows somewhat smaller value than in field. It implies that cores used in lab test are disturbed. Therefore, the shear strength measured in lab tests is smaller than in field.

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초 록

말뚝의 지지층으로 활용 가능한 풍화된 화강암 지반은 국내에서 빈번하게 나타나는 지반이다. 하지만 풍화된 화강암 지반은 그 특성이 일반적인 토사 지반이나 암반과 달라 불교란 시료의 채취가 어려워 강도 정수를 결정하는데 어려움이 있다. 따라서 본 연구는 국내 풍화된 화강암 지반의 강도 정수를 결정하기 위해 국내에서 일반적으로 활용되는 프레셔미터시험(PMT)과 공내전단시험(BST)을 적용하여 풍화된 화강암 지반의 강도 정수를 추정하고 실내 시험 및 기존 연구에서 제안된 풍화된 화강암 지반의 강도 정수와 비교하여 현장 시험의 타당성을 검증하고자 하였다. 비교적 교란이 적은 시료를 채취하기 위하여 굴진 비트와 지반 사이의 마찰을 줄여주는 폴리머를 사용하고 굴진 간격을 일반적인 경우(3 m)보다 짧게(1 m) 적용하였다. 실내 시험은 일축압축시험과 삼축압축시험을 적용하였으며 각 시험 과정에서 탄성파 속도를 측정하였다. 프레셔미터시험에는 Haberfield와 Johnston (1990)이 제안한 방법을 적용하여 강도 정수를 추정하였다. 프레셔미터시험, 공내전단시험, 실내역학시험, 그리고 기존 연구에서 제안된 강도 정수 사이에는 상당한 차이가 나타났으며 이를 비교 분석하여 풍화된 화강암 지반에 적합한 강도 정수 산출 방법에 대해 제안하였다.

주요어 : 풍화암, 강도 정수, 프레셔미터 시험, 공내전단시험

학 번 : 2016-21257